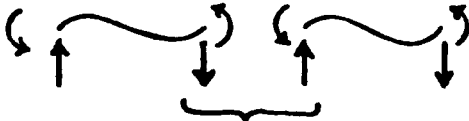
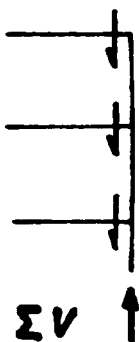


COLUMNS: SPECIAL TRANSVERSE REINFORCEMENT-CONT'D

SPECIAL TRANSV. REINF. IS ALSO REQUIRED WHERE COLUMN CAPACITY IS LESS THAN THE SUM OF THE SHEARS ABOVE. REF. APP. C PARAGRAPH C12



- ΣV IS THE COLUMN LOAD AT THIS LEVEL.
- AT INTERIOR COL'S THIS SUM IS RELATIVELY SMALL.
- END COLUMNS ARE USUALLY CRITICAL.



1. INCLUDE ALL BEAMS ABOVE THE COLUMN IN QUESTION.

$$2. V_{u_i} = \frac{(M_P^A + M_P^B)}{L} \Big|_{P.13} + V_{D+L} \Big|_{P.12}$$

3. AT THE COLUMN IN QUESTION, CALCULATE THE MAX. MOMENT TRANSFERRED TO THE COLUMN BY THE YIELDING BEAM.

4. DOES THE COLUMN HAVE THE CAPACITY TO CARRY ΣV_u WITH THIS BEAM MOMENT?
 YES: NO ADD'L REINF. REQ'D.
 NO: PROVIDE THE SPECIAL TRANSV. REINF.
 CALCULATED ON P.19
 FOR FULL HEIGHT.

SEE P. 21 FOR SAMPLE CALC.

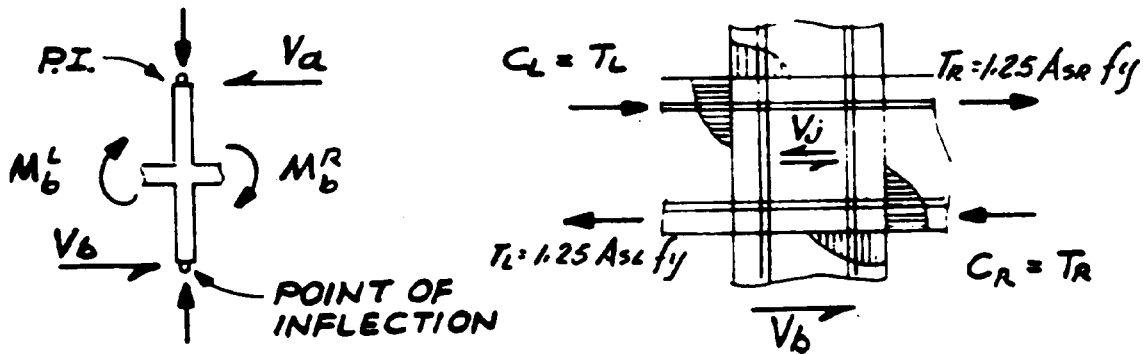
Figure D-2. Continued.

COLUMNS: SPECIAL TRANSVERSE REINFORCEMENT CONT.

FRAME COLUMN	B 1	
ROOF BEAM $\Sigma M_p/L$ V_{D+L} V_u^R	35 55 <hr/> 90	CALCS. NOT SHOWN
3RD FLR. BEAM $\Sigma M_p/L$ V_{D+L} V_u^R	43 67 <hr/> 110	ASSUME SAME AS 2ND
2ND FLR. BEAM $\Sigma M_p/L$ V_{D+L} V_u^R	43 67 <hr/> 110	$= \frac{794 + 489}{(p.12)^{30}} (p.13)$
ΣV_u ABOVE	310	
M_p FROM BM	397	$= \frac{1}{2} BM \quad M_p = \frac{794}{2}$
ALLOW COL. M WITH $P = \Sigma V_u$	659	(ACI. SP 17A VOL. 2 CHART E4-60-75)
COL. M > M_p	YES	
SPEC. TRANSV. REINF.	NO	

Figure D-2. Continued.

BEAM - COLUMN JOINT



FORCES ON COLUMN

FORCES ON BEAM COLUMN JOINT

THE JOINT SHEAR,
$$V_j = 1.25 A_{sr} f_y + C_L - V_a$$
$$= 1.25 (A_{sr} + A_{sl}) f_y - V_a$$
$$v_j = V_j / \phi A_j$$

$$A_j = \text{BEAM WIDTH} \times \text{COLUMN DEPTH} = 28" \times 24"$$

INTERIOR JOINTS: REF. ACI 21.6.2,3

JOINTS ARE CONFINED

$$\therefore v_j \leq (20 \sqrt{F'_c} = 1265 \text{ psi})$$

TRANSV. HOOP REINF. MAY BE $\frac{1}{2}$ OF COLUMN HOOP REINF.

EXTERIOR JOINTS:

JOINTS ARE NOT CONFINED

$$\therefore v_j \leq (15 \sqrt{F'_c} = 949 \text{ psi})$$

TRANSV. HOOPS SAME AS COL. HOOPS

Figure D-2. Continued.

BEAM-COLUMN JOINT-CONT.

FRAME (B)

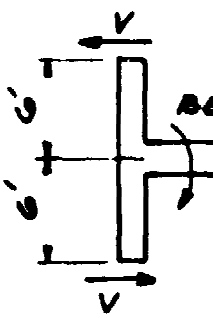
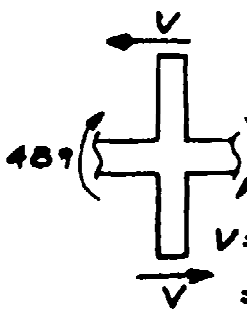
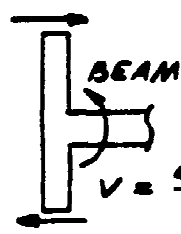
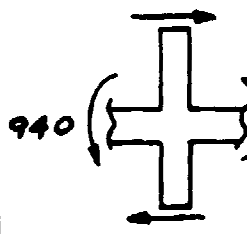
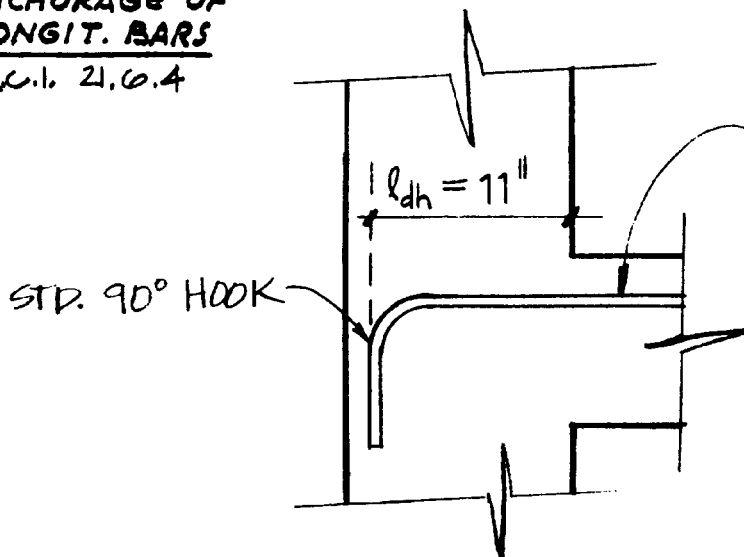
2ND STORY	COLUMN B-1	COLUMN B-2
<p><u>HOOPS IN JOINT</u></p> <p>← E</p> <p>BEAM A_{SR}</p> <p>BEAM A_{SL}</p> <p>$(A_{SR} + A_{SL})125 F_y$</p> <p>V_u (see Fig. 8-9)</p> <p>$v_u = \frac{V_u}{\phi b d}$</p>	 <p>BEAM $M_p = 794$ P.13</p> <p>$V = \frac{794 \text{ ft}}{12} = 66.2 \text{ k}$</p> <p>5.0 IN² (5-#9)</p> <p>0</p> <p>$(5.0 + 0)(75) = 375 \text{ k}$</p> <p>$375 - 66 = 309 \text{ k}$</p> <p>$\frac{309,000}{0.85 \times 28 \times 24} = 541 \text{ psi}$</p>	 <p>489 (←) 940</p> <p>$V = \frac{940 + 489}{12} = 119 \text{ k}$</p> <p>6.0 (6-#9)</p> <p>3.0 (3-#9)</p> <p>$(6.0 + 3.0)(75) = 675 \text{ k}$</p> <p>$675 - 119 = 556 \text{ k}$</p> <p>$\frac{556,000}{0.85 \times 28 \times 24} = 973 \text{ psi}$</p>
<p>→ E</p> <p>BEAM A_{SR}</p> <p>BEAM A_{SL}</p> <p>V_u (see Fig. 8-9)</p> <p>v_u</p>	 <p>BEAM $M_p = 489$ P.13</p> <p>$V = \frac{489}{12} = 40.8$</p> <p>3.0 IN²</p> <p>0</p> <p>$225 - 41 = 184 \text{ k}$</p> <p>332 psi</p>	 <p>940 (←) 489</p> <p>$V = 119$</p> <p>3.0 IN²</p> <p>6.0 IN²</p> <p>$675 - 119 = 556 \text{ k}$</p> <p>973 psi</p>

Figure D-2. Continued.

<u>BEAM-COLUMN JOINT-CONT.</u>		FRAME (B)
2ND STORY	COLUMN B-1	COLUMN B-2
Nj	541 < 949 p.s.i. O.K.	973 < 1265 p.s.i. O.K.
#4 HOOPS	SAME AS COL. (P. 19)	1/2 THAT OF COL. (P. 19)
HOOP SPACING	4"	8"

ANCHORAGE OF
LONGIT. BARS
AC.I. 21.6.4



EXTERIOR JOINT
OF COLUMN B-1.

#9 BARE
 $d_b = 1.128''$

$$l_{dh} = \frac{f_y d_b}{65 \sqrt{f'_c}}$$

$$= \frac{60,000 \times 1.128''}{65 \times \sqrt{4000}}$$

$$= 16.5''$$

NOTE: FOR INTERIOR JOINTS, COL.
DEPTH > $20d_b = 22.4''$
O.K.

Figure D-2. Continued.

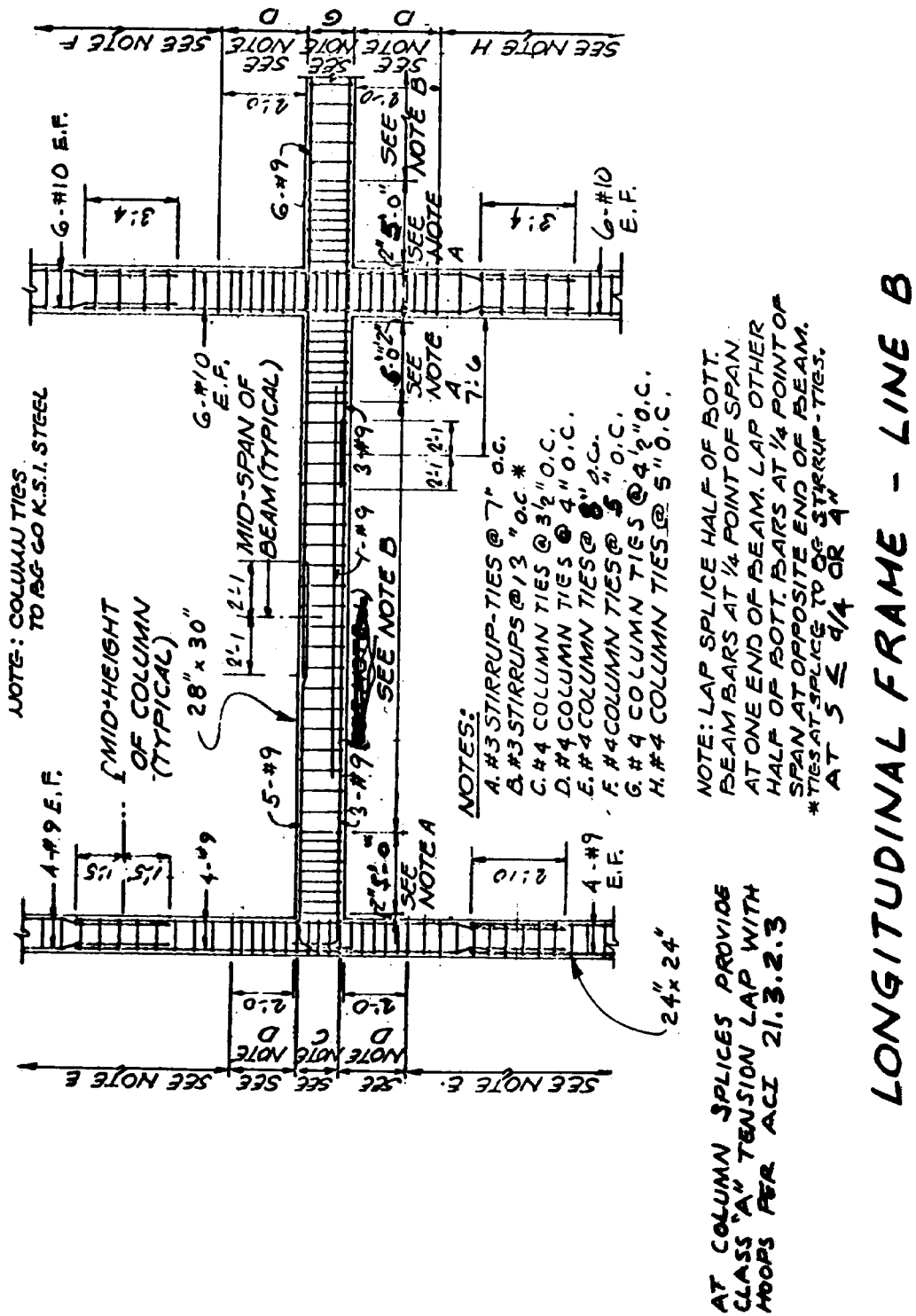


Figure D-2. Continued.

DESIGN EXAMPLE D-3

Steel Special Moment Resisting Frame and Steel Braced Frame

Description of Structure. A three-story Administration Building with transverse special moment-resisting frames and longitudinal concentric or eccentric braced frames in structural steel, using non-bearing, non-shear, exterior walls (skin) of flexible insulated metal panels. There are a series of interior vertical load-carrying column and girder bents in addition to the space frame. The structural concept is illustrated on Sheets 2 and 3.

Construction Outline.Roof:

Built-up 5 ply.
Metal decking with
insulation board.
Suspended ceiling.

2nd & 3rd Floors:

Metal decking with concrete fill.
Asphalt tile.
Suspended ceiling.

1st Floor:

Concrete slab-on-grade.

Exterior Walls:

Non-bearing, non shear,
insulated metal panels.

Partitions:

Non-structural removable
drywall.

Design Concept. The transverse ductile moment-resisting frames are independent of the longitudinal braced frames. The moment frames are designed to $R_w = 12$; the concentric braced frames to $R_w = 8$; the eccentric braced frames to $R_w = 10$. The metal deck roof system forms a flexible diaphragm; therefore the roof loads are distributed to the frames by tributary area rather than by frame stiffnesses. The metal deck with concrete fill system for the floors form rigid diaphragms and the seismic loads are proportioned to the frames by the frame stiffnesses.

Discussion. Because of the importance of drift of flexible frames, the example shows several stages of design. Preliminary design to find sizes by approximate methods, using different sets of forces for stress and drift. The resulting trial sizes are then used in a computer analysis. (The frames are simple enough to be calculated by hand, but the computer makes short work of calculating design forces, frame period and drift). Final design is discussed, and examples are given for modifications to the results of the computer analysis for accommodating various stress and deflection criteria with consistent sets of member sizes, period, design force, and drift.

Figure D-3. Steel ductile moment resisting space frame and steel braced frame.

LOADS.

ROOF:

5-PLY ROOFING	=	6.0 P.S.F.
1" INSULATION	=	1.5
STEEL DECK	=	2.3
STEEL PURLINS	=	3.7
STEEL GIRDERS	=	1.2
CEILING	=	10.0

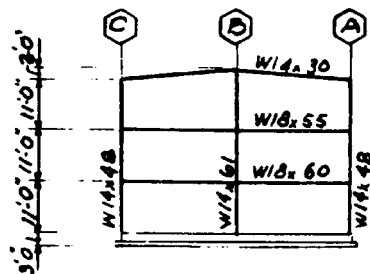
MISCELLANEOUS	=	1.0
DEAD LOAD	=	<u>25.7 P.S.F.</u>

ADD FOR SEISMIC:		
PARTITIONS	=	<u>10.0</u>
TOTAL FOR SEISMIC	=	<u>35.7 P.S.F.</u>

2ND & 3RD FLOORS:

FINISH	=	1.0 P.S.F.
STEEL DECK	=	3.1
CONCRETE FILL	=	32.0
STEEL BEAMS	=	5.9
STEEL GIRDERS		
& COLUMNS	=	1.6
PARTITIONS	=	20.0
CEILING	=	10.0
MISCELLANEOUS	=	1.0
DEAD LOAD	=	<u>74.5 P.S.F.</u>

LIVE LOAD	=	<u>50.0 P.S.F.</u>
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LINES 1, 4, & 7

TRANSVERSE SPECIAL MOMENT RESISTING FRAMES
SEE SHT 7

Figure D-3. Continued.

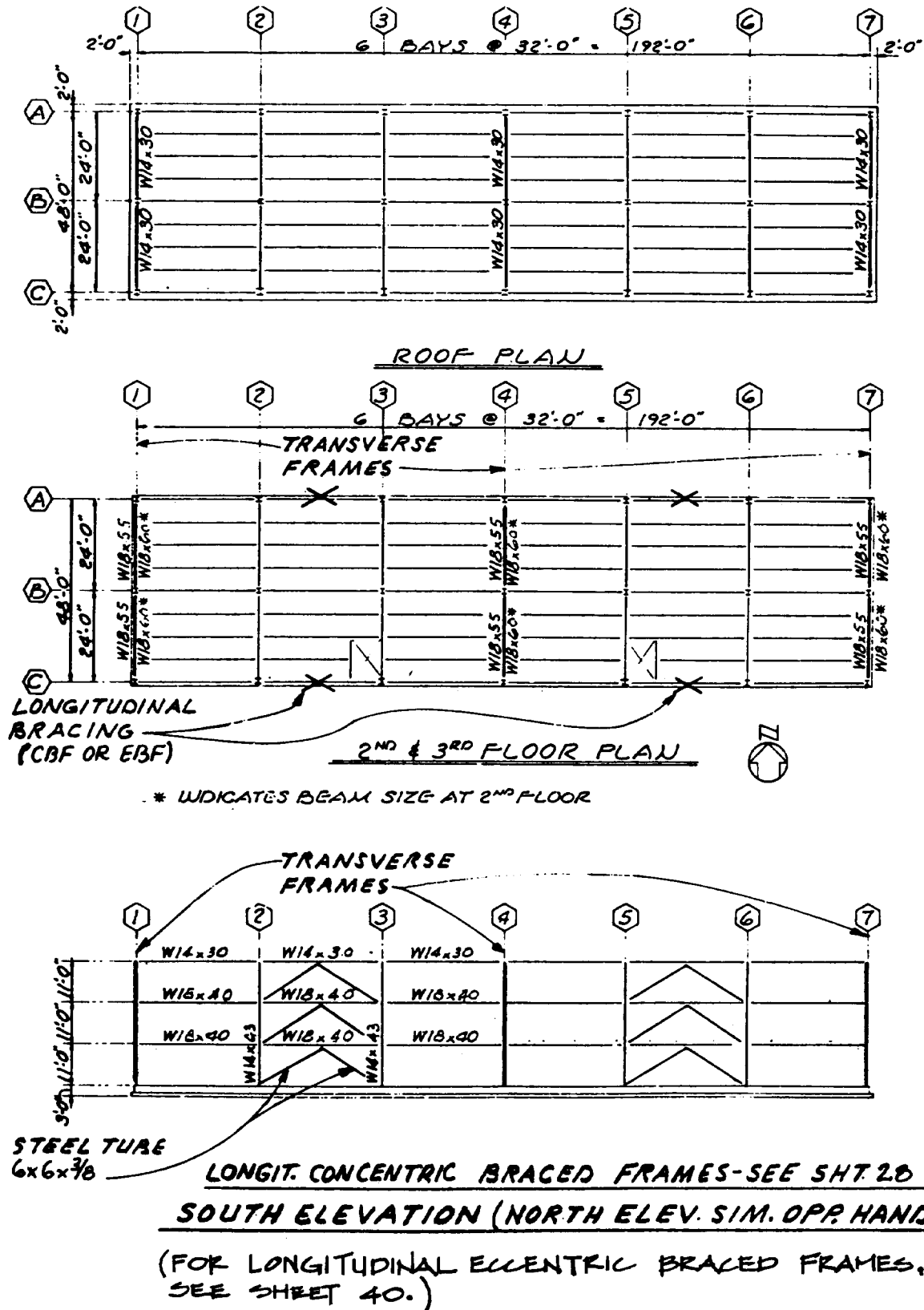


Figure D-3. Continued.

DESIGN PROCEDURE

Example Page

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- | | |
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| 10. Member Stresses | 21, 22 |
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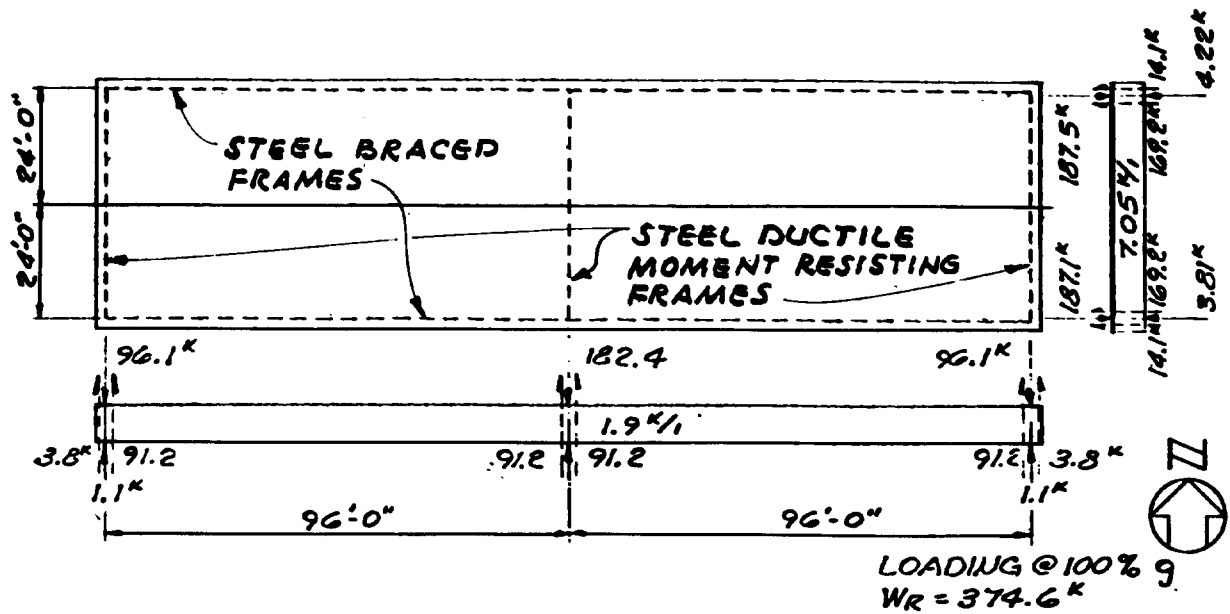
C. LONGITUDINAL CONCENTRIC BRACED FRAME

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- | | |
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Figure D-3. Continued.



LOADS FOR ROOF DIAPHRAGM

EXTERIOR WALLS (NON STRUCTURAL EXTERIOR COVERING)

WALL WT. $5.3 \text{ PSF} \times 5.5' = 29.0 \text{ \#/ft}$ FRACTION OF SOLID WALL-WINDOWS OUT

N. WALL = $29 \times .75 = 22 \times 192' = 4224 \text{ \#}$

S. WALL = $29 \times .68 = 19.8 \times 192' = 3801 \text{ \#}$
 41.8 \#/ft

WALL WT. $5.3 \text{ PSF} \times 6' = 31.8 \text{ \#/ft}$

E. WALL = $31.8 \times .76 = 24 \times 48' = 1.152 \text{ \#}$

W. WALL = $31.8 \times .76 = 24 \times 48' = 1.152 \text{ \#}$
 48 \#/ft

N-S LOADS

ROOF = $35.7 \times 52' = 1856.4$

WALLS = 41.8 \#/ft
 1898.2 \#/ft

E-W LOADS

ROOF = $35.7 \times 196' = 6997.2$

WALLS = 48.0 \#/ft
 7045.2 \#/ft

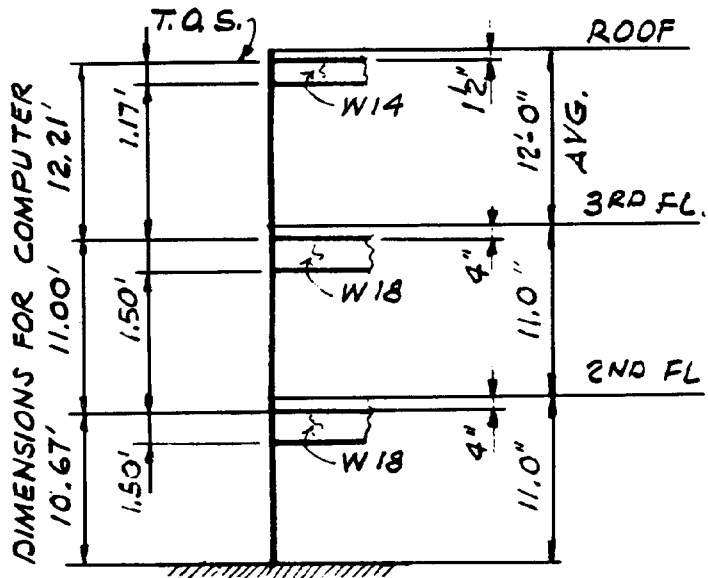
Figure D-3. Continued.

FLOOR WEIGHT FOR SEISMIC = 74.5 PSF
WALL WT. $5.3 \text{ }^{\circ}\text{SF} \times 11' = 58.3 \text{ }^{\circ}/$
N. WALL = $58.3 \times .75 = 44 \times 192' = 8448^{\circ}$
S. WALL = $58.3 \times .70 = \underline{39.6} \times 192' = 7603^{\circ}$
83.6 %
E. WALL = $58.3 \times .75 = 44 \times 48' = 2112^{\circ}$
W. WALL = $58.3 \times .75 = \underline{44} \times 48' = 2112^{\circ}$
88 %

$$\begin{array}{r} \text{FLOOR} = 74.5 \times 48' = 3576.0 \\ \text{WALL} = \quad \quad \quad \underline{84.0} \\ \quad \quad \quad 3660.0 \text{ \#} \end{array}$$

FLOOR = $74.5 \times 192' = 14304.$
WALL 88.
14392. #/

D-78

TRANVERSE (N-S) DIRECTION:STEEL SPECIAL MOMENT-RESISTING FRAMES

THE COLUMN BASE IS ASSUMED FIXED.

THIS IS NOT ALWAYS FEASIBLE, ACTUAL FOUNDATION CONDITIONS SHOULD BE CAREFULLY STUDIED, AND REALISTIC ASSUMPTIONS SHOULD BE MADE FOR ANALYSIS.

FRAME CHARACTERISTICS

Gravity load: The middle frame will take twice as much gravity loads as each end frame according to tributary area.

Seismic load: All three frames will have the same proportions. Assuming the roof diaphragm is flexible, and using the tributary area approach, the middle frame will take half of the seismic load at the roof level while each end frame will take one quarter. Assuming the floor diaphragms are rigid, the third floor diaphragm will distribute some of the lateral load that originates at the roof level from the middle frame to the end frames; also, because the three frames have equal stiffness, the rigid floor diaphragms will distribute one third of the load that originates at each floor to each of the three frames. The roof diaphragm is not fully flexible: the middle frame will take something less than half and the end frames something more than one quarter each of the roof load. Also the floor diaphragms are not fully rigid: the end frames will probably not get a full third of the load. The example assumes that the middle frame keeps full half of the roof load and one third of the floor loads: what is probably an excessive load from the roof tends to offset what is probably a deficient load from the floors.

Figure D-3. Continued.

FRAME CHARACTERISTICS - cont'd

Because of accidental torsion the end frames will take some torsional forces below the third floor.

The total seismic forces being nearly the same in all frames, the design will be governed by the middle frame which takes twice as much gravity load as each end frame, and the design example will be concerned with this frame, i.e., the transverse frame on Line 4.

BUILDING PERIOD

In order to calculate lateral forces for design of the frame, the building period is needed. SEAOC provides two methods, Method A and Method B, and this example will make use of a third method, the Drift Limit Method.

Method A provides a simple formula based on the height of the building and the structural system, so it could be used as a first approximation for a preliminary design. Using $C_t = 0.035$ for steel frames and $h_n = 34$ feet, SEAOC eq 1-3 provides $T = 0.035 (34)^{3/4} = 0.49$ seconds. Method A is intentionally conservative; it tends to be a lower bound. This is particularly noticeable with steel moment frames. It would be desirable to use a longer period in order to reduce the design forces in a more realistic representation of the building; however this must be done with care because if the period is too long the preliminary design will be undersized. The code provides a limit on T by not allowing a value of C less than 80% of the value obtained by using T from Method A.

Method B is an accurate method, but it requires frame deflections which can be calculated only after a preliminary design is established.

The Drift Limit Method provides a period based on the assumption that the frame is so limber that it is at its maximum allowable deflection under code-prescribed loads. This provides an upper-bound period in contrast to the lower-bound period of method A. The period of a frame can be approximated by the formula $T = 2\pi\sqrt{\delta_n/a_n}$, where δ_n is the lateral roof displacement for the peak roof acceleration a_n . For the

Figure D-3. Continued.

BUILDING PERIOD - cont'd

equivalent static force procedure, δ_n is the roof displacement due to the prescribed forces, and a_n is approximately equal to $(1.7 V/W)g$, or $(1.7 ZIC/R_w)g$, where $C = 1.25S/T^{2/3}$ and g is the acceleration due to gravity. The formula can be written

$$T = 0.66 \left(\frac{\delta_n R_w}{ZIS} \right)^{3/4} \quad \text{with } \delta_n \text{ in feet.}$$

For $T > 0.7$ sec., the story drift limit is $0.03/R_w$. If the deflected shape is a straight line, $\delta_n = 0.030 h_n/R_w$. But it is not likely that the deflected shape will be a straight line. Let us assume that the average story drift is 0.80 times the maximum story drift; then, $\delta_n = 0.024 h_n/R_w$ and

$$T = 0.040 \left(\frac{h_n}{ZIS} \right)^{3/4} \quad \text{for } T > 0.7 \text{ sec.}$$

For $T < 0.7$ sec., the story drift limit is $0.04/R_w$ and

$$T = 0.050 \left(\frac{h_n}{ZIS} \right)^{3/4} \quad \text{for } T < 0.7 \text{ sec.}$$

The example will make use of this method. With $h_n = 34$, $Z = 0.4$, $I = 1.0$, and $S = 1.5$, $T = 0.83$ sec. ($T > 0.7$ sec.)

Period calculations, being based on framing members, are "bare-frame" periods, that is, they do not account for the participation of nonseismic frames and nonstructural elements. For force calculations, the calculated period will be reduced in order to account for the stiffening effects of these frames and other elements. For this example, we will divide the drift limit period of 0.83 sec. by a factor of 1.2, a number obtained by judgment.

The design lateral forces will be based on a "whole building" period of $T = 0.83/1.2 = 0.69$ sec.

Note that with $T_A = 0.49$ sec, $C_A = 2.75$. With $T = 0.69$ sec, $C = 2.40$ which is greater than $0.80 \times 2.75 = 2.20$.

Figure D-3. Continued.

LATERAL FORCES FOR PRELIMINARY DESIGN

USE $T = 0.69$ SEC. FOR STRESS ANALYSIS.

BUILDING: A-3 $T = 0.69$ SEC.
 $F_T = 0.07 TV = 0^*$

DIRECTION: TRANSVERSE
 $F_x = (V - F_T) \frac{Wh}{\sum Wh} = 1.0 V \frac{Wh}{\sum Wh}$
 $Z = 0.4; I = 1.0 R_w = 12$
 $V = \frac{ZIC}{R_w} W = 0.080 W$
 $C = \frac{1.25s}{T^{2/3}} = \frac{1.25 \times 1.5}{(0.69)^{2/3}} = 2.40$
 $= 143$ KIPS

$*F_T = 0$ WHEN $T \leq 0.7$ SEC.

$W = 1789$ KIPS

LEVEL	h FT (2)	Δh FT (3)	W KIPS (4)	Σ W (5)	(2) × 4 Wh (6)	$\frac{Wh}{\sum Wh}$ (7)	F (8)	Σ (7) V KIPS (9)	(3) × (9) ΔOTM K-FT (10)	Σ (10) OTM K-FT (11)	(6) × (4) $\frac{F}{W}$ (12)	(9) ÷ (5) $\frac{V}{\sum W}$ (13)
R	34		375	375	12,750	0.35	$F_T = 0$				0.133	0.133
3	22	12	707	1082	15,554	0.43	50.0	50	600	600	0.087	0.103
2	11	11	707	1789	7,777	0.22	61.5	111.5	1226	1826	0.045	0.080
		11					31.5	143.0	1573	3399		*
Σ			1789		36,081	1.00	143.0					*

** ALL < 0.14 , ∴ USE 0.14
FOR DIAPHRAGMS.
(SEAOC 1 H 2 j)

Figure D-3. Continued.

DISTRIBUTION OF FORCES TO FRAMES

Since the roof diaphragm is relatively flexible, the roof forces are distributed by tributary area.

The 2nd and 3rd floor diaphragms distribute the floor forces to the frames according to their relative rigidities.

The transverse frames on lines 1, 4 and 7 are alike, and for preliminary design we may take their rigidity proportional to

$$K_1 = \frac{1/3(\text{BASE SHEAR})}{\text{DRIFT}} = \frac{1/3(143)}{0.0025(34')} = 561 \text{ k/ft}$$

← see page 10
← see page 9

The longitudinal frames on lines A and C have a rigidity based on preliminary trials:

$$K_A = \frac{1/2(\text{BASE SHEAR})}{\text{DRIFT}} = \frac{1/2(250)}{0.30''/12} = 5000 \text{ k/ft}$$

← see page 27
← prelim calcs (not shown)

Use Rel. $K_1 = 1$ and Rel. $K_A = \frac{5000}{561} = 8.91$, say 9

Because of symmetry there is no "calculated" torsion. The "accidental" torsion is the story force, F , times the nominal eccentricity of 5% of the max. building dimension: perpendicular to the direction of force under consideration:

FOR forces in
the transverse
direction.

$$M_t = F_{\text{transv.}} \times 0.05 \times 192' = 9.6 F_{\text{transv.}}$$

$$\text{TORSIONAL SHEAR} = \frac{Kd}{\Sigma Kd^2} 9.6 F_{\text{transv.}}$$

FOR forces in
the longitudinal
direction.

$$M_t = F_{\text{long}} \times 0.05 \times 48 = 2.4 F_{\text{long.}}$$

$$\text{TORSIONAL SHEAR} = \frac{Kd}{\Sigma Kd^2} 2.4 F_{\text{long.}}$$

$$= \frac{216}{28,800} \times 2.4 F_{\text{long}} = 0.02 F_{\text{long}}$$

Figure D-3. Continued.

DISTRIBUTION OF FORCES - CONT.

FRAME	REL K	d	Kd	Kd ²	DIRECT SHEAR	TORSIONAL SHEAR	DESIGN SHEAR
1	1	96	96	9216	.33 F _T	± .03 F _T	.36 F _T
4	1	0	0		.33 F _T	0	.33 F _T
7	1	96	96	9216	.33 F _T	± .03 F _T	.36 F _T
192							
A	9	24	216	5184	.50 F _L	± 0.02 F _L	0.52 F _L *
C	9	24	216	5184	.50 F _L	± 0.02 F _L	0.52 F _L
			432				
				Σ = 28,800			

*THESE WILL BE USED FOR DESIGN OF THE LONGITUDINAL FRAMES. SEE P. 28.

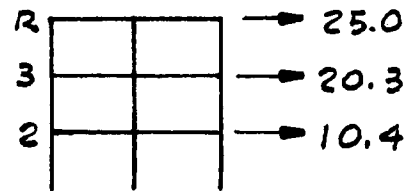
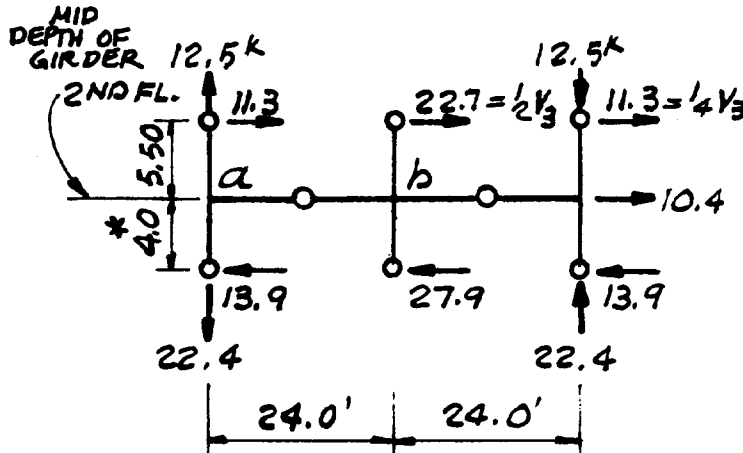
DISTRIBUTION TO TRANSVERSE FRAMES

FRAME	1	4	7
<u>ROOF</u> (BY TRIBUTARY AREA)			
50.0	x .25 = 12.5	x .50 = 25.0	x .25 = 12.5
<u>3RD.</u> (BY REL. RIGIDITY)			
61.5	x .36 = 22.1	x .33 = 20.3	x .36 = 22.1
<u>2ND.</u>			
31.5	x .36 = 11.3	x .33 = 10.4	x .36 = 11.3
143.0 ^K	45.9 ^K	55.7 ^K	45.9 ^K

Figure D-3. Continued.

PRELIMINARY DESIGN

MEMBER FORCES BY PORTAL METHOD - FRAME 4



FRAME FORCES

EXTERIOR COLUMN, (MOM. AT
 & GIRD.)

$$\text{ABOVE } a, M = 11.3k \times 5.50' = 62.2$$

$$\text{BELOW } a, " = 13.9k \times 4.0' = 55.6$$

$$\underline{117.8k'}$$

INTER. COLUMN (MOM. AT
 & GIRD)

$$\text{ABOVE } b, M = 22.7 \times 5.50 = 124.9$$

$$\text{BELOW } b, " = 27.9 \times 4.0 = 111.6$$

$$\underline{236.5k}$$

GIRDER (MOM. AT & COL.)

$$M_a = 117.8 \quad M_b = \frac{236.5}{2} = 118.3$$

$$V = \frac{117.8 + 118.3}{24'} = 9.84k$$

* ESTIMATED LOCATION OF
 INFLECTION CONSIDERING
 FIXITY OF BASE.

AT UPPER POINT OF

INFLECTION, **

$$M = (25.0k \times 18.6') + (20.3k \times 6.7')$$

$$= 601k'$$

$$\text{AXIAL} = \frac{601k'}{48} = 12.5k$$

AT LOWER P. I.

$$M = 601 + (45.3 \times 9.5) + (10.4 \times 4.0) = 1073k'$$

$$\text{AXIAL} = \frac{1073}{48} = 22.4k$$

** 18.6' IS APPROXIMATE
 DISTANCE FROM ROOF
 TO POINT OF INFLECTION.
 6.6' IS APPROXIMATE
 DISTANCE FROM TOP
 OF 3RD FLOOR SLAB
 TO POINT OF INFLECTION.

Figure D-3. Continued.

PRELIMINARY DESIGN - CONT.

FRAME 4

INTERIOR COLUMN

D. VERTICAL LOAD ON CENTER FRAME

$$\begin{aligned} \text{ROOF DL:} & 0.0257 \text{ KSF} \times 24' \times 32' = 19.8 \\ \text{3RD FLR. DL+LL} & = (0.0745 + 0.021) \times 24 \times 32' = 73.3 \\ \text{2ND FLR.} & \text{RED. LL} \checkmark = 73.3 \\ & W = 166.4 \text{ K} \end{aligned}$$

BY SYMMETRY, $M = 0$

b. SEISMIC LOAD, FROM P. 13

$$\begin{aligned} P & = 0 \\ M & = 27.9 \text{ K} \times (4.00 - 0.75) = 90.7 \text{ K-ft AT FACE OF GIRDER} \end{aligned}$$

C. VERTICAL + SEISMIC

$$\begin{aligned} P & = 166 + 0 = 166 \text{ K} \\ M & = 0 + 90.7 = 90.7 \text{ K} \end{aligned}$$

USE AISC 9TH EDITION, P. 3-9

$$\text{TRY } W14 \times 61, \quad \text{P. 3-24} \quad B_x = 0.194$$

$$P_{\text{EQUIV.}} = \frac{166 \text{ K} + 0.194(90.7 \times 12^{1/1})}{1.33} = 284 \text{ K}$$

$$\text{FOR } h' = 9.5' \quad \frac{W14 \times 61}{I = 640} \quad \text{ALLOWS } 334 \text{ K}$$

EXTERIOR COLUMN

VERTICAL + SEISMIC

$$\begin{aligned} P & \approx 166/2 + 22.4 = 105 \text{ K} \\ M & \approx 50 \text{ K-ft (est)} + 90.7/2 = 95 \text{ K-ft} \end{aligned} \quad \left. \vphantom{\begin{aligned} P & \approx 166/2 + 22.4 \\ M & \approx 50 \text{ K-ft (est)} + 90.7/2 \end{aligned}} \right\} P_{\text{EQUIV.}} = 246$$

$$\frac{W14 \times 48}{I = 485} \quad \text{ALLOWS } 246 \text{ K}$$

USE 14" COLUMNS FOR CONTROL OF DEFLECTIONS AND USE THE SAME SECTION FULL HEIGHT.

Figure D-3. Continued.

PRELIM. DESIGN - CONT.

FRAME 4

GIRDER - 2ND FLOOR

VERTICAL LOAD AT CENTER COLUMN

$$R = 0.08 \times 32 \times 24 = 61.4\% \text{ OR } 23.1 (1 + 74.5/50) = \underline{57.5\%}$$

$$\text{RED. LL} = 0.425 \times 50 = 21 \text{ PSF} \quad (1 - 0.575 = 0.425)$$

$$W_{D+L} = (0.0745 + 0.021) \times 32' = 2.38 + 0.67 = 3.05 \text{ K/1}$$

$$W_{D+L} = 3.05 \times 24' = 73.2 \text{ K}$$

$$M \approx \frac{WL}{12} = \frac{73.2 \times 24}{12} = 146.4$$

$$\text{SEISMIC} \quad M = \underline{118.3}$$

$$\text{VERT. + SEISMIC} \quad M = 265$$

USE AISC BEAM CHART, P. 2-166, 9TH ED, WITH

$$M = \frac{265}{1.33} = 199 \text{ K'}, \text{ AND UNBRACED LENGTH OF 6'}$$

FOR NEG. BENDING

$$\frac{W18 \times 60}{I = 986} \quad \text{ALLOW } 216 \text{ K'}$$

Figure D-3. Continued.

FRAME 4

VERTICAL LOAD - SAME AS 2ND, $M = 146 \text{ K}'$
 SEISMIC $V_R = 25.0$ — 6.25 K TO EXT. COL.
 12.5 K TO INT. COL.

3RD FLR. 12.5 5.50 6.20 22.7 $P. 13$

$\Sigma \text{ COL. } M = (12.5 \times 6.2') + (22.7 \times 5.5)$
 $= 202 \text{ K}'$

GIRDER $M = \frac{202}{2} = 101 \text{ K}'$

VERT. + SEISMIC = $M = 146 + 101 = 247 \div 1.33 = 186 \text{ K}$

W18x55 ALLOW 197

VERTICAL LOAD

$$ROOF DL + LL = 32'(0.0257 + 0.020) = 0.82 + 0.64 = 1.46 \text{ K/ft}$$

$$W_D = 0.82 \times 24' = 19.7 \text{ K}$$

$$M = \frac{19.7 \times 24'}{12} = 39.4 \text{ K'}$$

SEISMIC



$$GIRDER M = \frac{12.5 \times 6.04}{2} = 37.8$$

$$VERT. + SEISMIC = M = 39.4 + 37.8 = 77.2 \div 1.33 = 58.0 \text{ K'}$$

W14x22 ALLOW 58 OK FOR STRESS

USE W14x30 ALLOW 83 USE WIDER FLANGE FOR BETTER DETAILS

Figure D-3. Continued.

CRITERIA FOR FINAL DESIGN - FRAME 4

1. BUILDING PERIOD

- a. For calculating frame design forces, use the whole building period estimated as $T = 0.69$ sec. (p. 9)
- b. For calculating drift, use the bare frame period. This will be obtained from the computer analysis or from the use of Method B. Method B will give a frame period of 0.83 sec.

2. DESIGN FORCES

As indicated item 1a above, use the building period of $T = 0.69$ sec. and the associated base shear of 143^k and frame shear of 55.7^k (p. 12). This is the input for the computer analysis (p. 18).

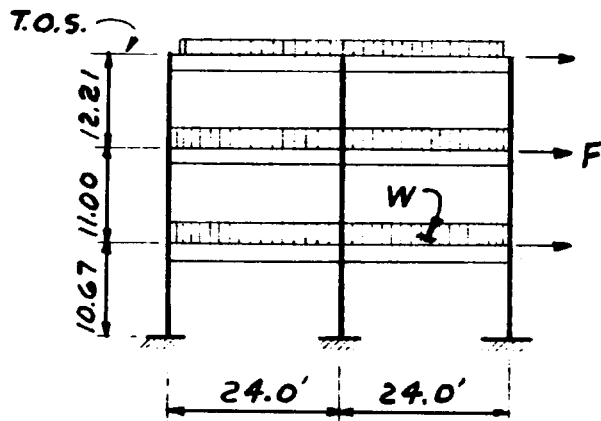
3. DEFLECTIONS

The deflections obtained from this analysis, based on whole building forces, will be modified for the calculation of drift under bare-frame forces (p.20).

Figure D-3. Continued.

FRAME ANALYSIS — FRAME 4

COMPUTER INPUT



KIPS, FEET, SECONDS

RIGID FRAME.

STEEL: $E = 4,176,000 \text{ KSF}$

COLUMN BASES FIXED.

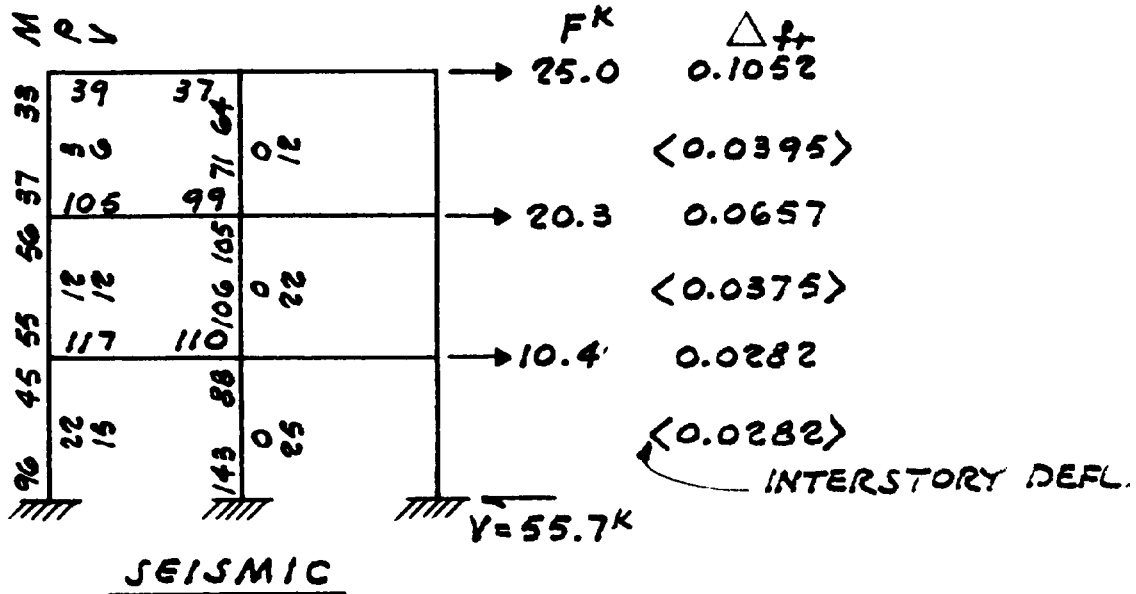
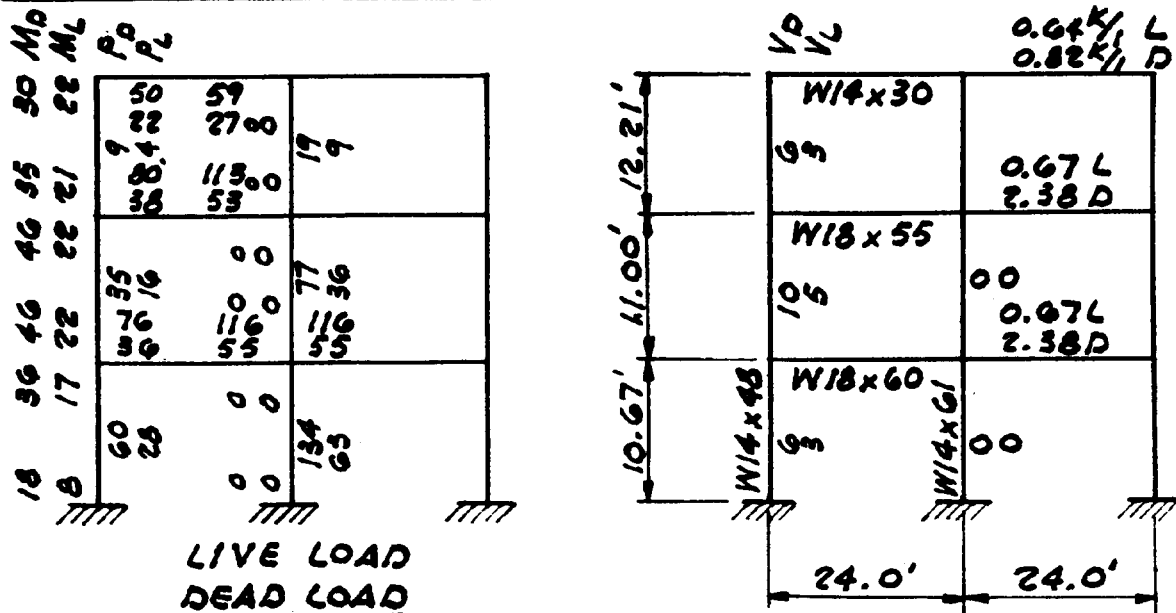
	EXT. COL.	INT. COL.
SIZE	W14x48	W14x61
I	0.0234	0.0309
A	0.0979	0.1243
A _w	0.0325	0.0365

LEVEL	GIRDER						TRIB W	MASS = W/9	LATERAL FORCE
	SIZE	I	A	A _w	W _{DL}	W _{LL}			
R	W14x30	0.0140	0.0613	0.0260	0.82	0.64	187 ^K	5.81	25.0
3	W18x55	0.0430	0.1125	0.0491	2.38	0.67	233	7.24	20.3
2	W18x60	0.0476	0.1229	0.0527	2.38	0.67	233	7.24	10.4
DATA FROM PAGE					14 & 15	5 & 6			11

TRIB. ROOF WT. IS $\frac{1}{2}$ OF TOTAL SINCE DIAPH. IS FLEXIBLE
 TRIB. FLOOR WT. IS $0.33 \times \text{TOTAL}$ ACCORDING TO REL. RIG. OF
 FRAMES. $g = 32.2 \text{ FT./SEC.}^2$

Figure D-3. Continued.

FRAME ANALYSIS - CONT. FRAME 4 **COMPUTER OUTPUT**



$$RIGIDITY: K = \frac{V}{\Delta_R} = \frac{55.7}{0.1052} = 529 \text{ K/ft}$$

Figure D-3. Continued.

FINAL DESIGN - CONT. FRAME 4 - DRIFT

Before proceeding with the detailed final design, we will check the drift.

From the computer analysis, the maximum drift is 0.0395 ft. (p. 19). This is based on the whole building period $T = 0.69$ sec. and the frame shear of 143^k (p. 10).

For the drift check, use the deflection associated with the bare frame period $T = 0.83$ sec. (p. 9).

$$C = (1.25 \times 1.5) / (0.83)^{2/3} = 2.12$$

$$CS = ZIC/R_w = (0.4 \times 1.0 \times 2.12) / 12 = 0.071$$

$$\text{Base Shear} = 0.071 \times 1789 = 127^k$$

Multiply deflections from frame analysis by the ratio $127/143 = 0.89$.

$$\text{Maximum drift} = 0.89 \times 0.0395' = 0.035'$$

$$\text{Allowed drift} = (0.03/12) \times 12' = 0.030' \quad (T = 0.83 > 0.7)$$

The frame drift is **17%** over the limit. It should be stiffened. There are three options:

- (1) increase the member sizes
- (2) use more than three frames
- (3) make the roof diaphragm rigid

We will change the interior column to W14x68 and proceed with the detailed design. Further changes may be necessary, and we will make a final drift check after other checks are completed.

Note that the assumed condition of fixed columns bases is difficult to achieve. If the bases are not fully fixed, the frame will be more flexible than assumed, and the frames would have to be further stiffened.

Figure D-3. Continued.

FINAL DESIGN - CONT. FRAME 4
MEMBER STRESSES

(1) SAMPLE CALCULATION FOR 2ND FLR. GIRDER

	M_D	M_L	M_E	M_{D+L}	$\frac{M_{D+L+E}}{1.33}$
AT EXT. COL.	76	36	117	112	172
AT INT. COL.	116	55	110	171	211

DES. $M = 211 \text{ K'}$ UNBRACED LENGTH = 6'

W18 x 60 ALLOW $M = 216 \text{ K'}$ AISC 9TH ED., P. 2-166

PROVIDE GIRDER BRACING PER SEAOC 4FB:

MAX UNBRACED LENGTH = $96 r_y$

$$= \frac{96 (1.69)}{12} = 13.5 \text{ FT.} > 6 \text{ FT.}$$

BRACING IS ADEQUATE

(2) SAMPLE CALCULATION FOR COLUMN,
 SEE NEXT PAGE.

Figure D-3. Continued.

FINAL DESIGN - CONT. - FRAME 4 MEMBER STRESSES - CONT.

2.) SAMPLE CALCULATION FOR COLUMNS FIRST STORY AT BASE

$K_y = 1.0$ (COLUMNS ARE BRACED
BY BEAMS)

USE $K_x = 1.0$ PER
SEAOC 403

COL. SHOULD ALSO BE CHECKED
FOR SEAOC 403. THAT
PROVISION WILL NOT GOVERN
IN THIS EXAMPLE, BUT WILL
BE ILLUSTRATED IN THE
BRACED FRAME EXAMPLE

FRAME 4		EXT.		INT.	
MOMENT.		W14x48		W14x68	
		A=14.1 S=70.2		A=20.0 S=103	
FOR COLUMNS		P	M	P	M
DEAD	D	60	18	134	0
	L	28	8	63	0
	D+L	88	26	197	0
	E	22	96	0	143
→	D+L+E	83	92	148	108
	1.33				
	f _a	5.89 KSI		7.40 KSI	
	f _b	15.7 KSI		12.6 KSI	
	K _x	1.0		1.0	
	l	119"		119"	
	r _x	5.85"		6.01	
	Kl/r _x	20.3		19.8	
	P _a	28.3		28.3	
	K _y	1.0		1.0	
	l	119		119	
	r _y	1.91		2.46	
	Kl/r _y	62.3		48.6	
	→ F _a	22.3		24.6	
		F' _{ex}	362		381
	F _{bx}	33.0		33.0	
1 } x(f _b /F _b)	f _a /F _a	0.264		0.301	
	f _b /F _b	0.476		0.382	
		0.411		0.331	
		0.675		0.632	
		0.672		0.629	
=					
SUMMATIONS < 1.0 (OK)					

Figure D-3. Continued.

GIRDER-COLUMN CONNECTION

Girder data W18x60 Gr. 36

Girder strength in flexure

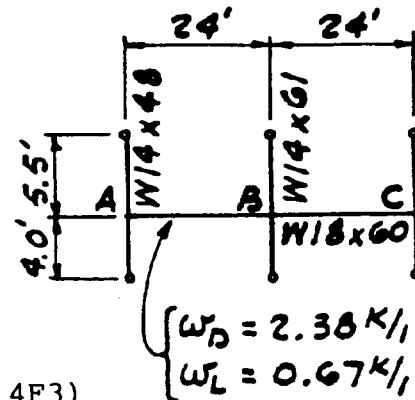
This is $M_s = ZF_y$ (SEAOC 4C2).
This is the same as plastic moment M_p
given in the in the AISC
Plastic Design Selection Table:

$$M_p = 369 \text{ k'}$$

Girder stability W18x60

$$b/2t_f = 5.44 < 52/\sqrt{36} = 8.66 \text{ (SEAOC 4F3)}$$

$$d/t_w = 43.5 \leq 412/\sqrt{36} = 68.7 \text{ (AISC Ch.N)}$$



Requirements for girder-column connection

SEAOC 4F1a requires development of the lesser of the strength of the girder in flexure (M_s) and the moment associated with the panel zone shear strength. The requirement of this manual is to develop the strength of the girder in flexure. This is accomplished according to SEAOC 4F1b.

Girder flange connection to column

Provide full butt-weld connections of the flanges to the columns. (SEAOC 4F1b(1))

Girder web connection to column

Design shall be based on the gravity loads plus the seismic load associated with compliance with SEAOC 4F1a. (SEAOC 4F1b(2))

$$\begin{aligned} \text{Vert. shear: } V_G &= (2.38+0.67)24'/2 + (171-112)/2 = 39.1 \\ \text{Seismic shear} &= 2M_s/L = 2 \times 369 \text{ k'}/22.83' = 32.3 \\ \text{Design } V &= 71.4 \text{ k} \end{aligned}$$

The method of developing the flexural capacity of the web depends on the relative size of the flanges, i.e., whether the flange strength ($bt_f(d-t_f) \times F_y$) is greater or less than 70% of the total strength ($0.7Z_x F_y$)

Figure D-3. Continued.

GIRDER-COLUMN CONNECTION

Girder web connection to column - cont.

$$b = 7.555"; t_f = 0.695"; d = 18.24"; Z_x = 123 \text{ in}^3$$

For 70% of entire section:

$$0.7 Z_x = 0.7 \times 123 = 86 \text{ in}^3$$

For the flanges alone:

$$b t_f (d - t_f) = (7.555") (0.695") (18.24 - 0.695") = 92.12 \text{ in}^3$$

As $92.12 > 86$, the connection can be made by welding and/or high-strength bolting according to (SEAONC 4F1b(2)(a))

If the chosen girder had had flange strength less than 70% of the total strength, the conventional web connection would have had to be supplemented with additional welding (to the web at the top and bottom of the shear tab on the column) according to SEAOC 4F1b(2)(b).

Girder web connection design

Assume 1" A325-SC bolts are selected,

$$\begin{aligned} \text{Use 4 bolts: Bolt Strength, } V &= 4 \times 1.7 \times 13.7 \\ &= 93.2 \text{ k} > 71.4 \end{aligned}$$

Shear plate:

$$\begin{aligned} Z &= 0.3125 (12.5)^2 / 4 = 12.2 \text{ in}^3 \\ f &= 71.4 \text{ k} \times 2" / 12.2 = 11.7 \text{ ksi} < 36 \\ v &= 71.4 / (0.3125 \times 12.5) = 18.3 \text{ ksi} < 0.55 \times 36 = 19.8 \end{aligned}$$

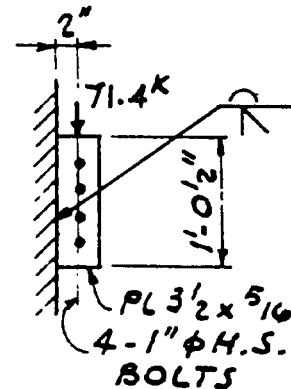


Figure D-3. Continued.

GIRDER-COLUMN CONNECTION

Girder-column panel zone -- Sample calc. for Joint B

Panel zone strength (SEAOC 4F2)

This is the moment corresponding to the development of the panel zone shear strength. This moment shall be the moment due to gravity loads plus 1.85 times seismic loads but need not exceed 0.8 times the summation of M_s at the beams.

Calculate moment arm between girder flanges:

$$d - t_f = 18.24 - 0.69 = 17.55"$$

Panel zone shear:

Gravity + 1.85 x seismic

One side of column, with D + L:

$$M_{D+L} = 171; \quad 1.85 M_E = 1.85 \times 110 = 204$$

Other side, with D:

$$M_D = -116; \quad 1.85 M_E = 204$$

$$\text{Sum of girder moments} = 171 + 204 - 116 + 204 = 463$$

0.8 M_s

$$\text{Sum of girder moments} = 2 \times 0.8 \times 369 = 590 \text{ k'}$$

Use panel zone strength associated with 463 k'.

(See SEAOC 4F10 for Drift calculations.)

Shear

$$\begin{aligned} \text{One side, top flange force} &= (171 + 204)(12)/17.55'' \\ &= 256 \text{ K} \end{aligned}$$

$$\begin{aligned} \text{Other side, top flange force} &= (-116 + 204)(12)/17.55'' \\ &= 60 \text{ K} \end{aligned}$$

Column shear above joint:

$$\text{sum girder moments} = 463$$

$$\text{column height} = 4.0 + 5.5 = 9.5'$$

$$\text{column shear} = 463 / 9.5 = 49 \text{ k}$$

$$\text{Shear} = 256 + 60 - 49 = 267 \text{ K}$$

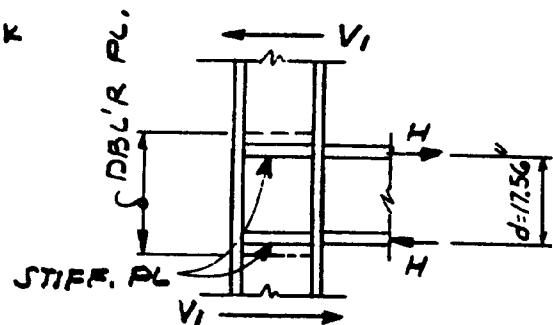


Figure D-3. Continued.

GIRDER-COLUMN CONNECTION -- Joint B -- cont.

Panel zone thickness

For thickness, t' , the panel zone can develop

$$V_j = 0.55 F_y d t' \quad (\text{modified SEAOC formula 4-1})$$

where t' is the effective thickness which consists of the combined thickness, t , of the web and doubler plate modified by the contribution of the the columns flanges (see below)

For W14 x 68 Grade 50 column

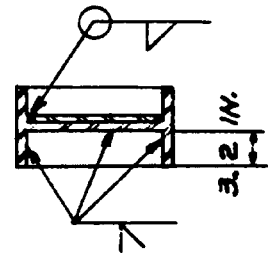
$$V_j = 0.55 (50) (14.04) t' = 386 t' \text{ kips}$$

For joint shear of 235 k,

$$\text{req'd } t' = 267 / 386 = 0.69"$$

$$\begin{aligned} t' &= t [1 + 3 b_c t_c^2 / d_p d_c t] \\ &= t [1 + 3 (10.03) (0.72)^2 / (18.24) (14.04) t] \\ &= t [1 + 0.061 / t] = t + 0.061 \end{aligned}$$

$$\begin{aligned} 0.69" &= t + 0.06 \\ t &= 0.63" \end{aligned}$$



For column web thickness of 7/16" req'd thickness of doubler plate = $0.63" - 7/16 = 0.19"$

Use a 1/4" minimum Grade 50 doubler plate, or consider using a column with a thicker web.

Check SEAOC 4F2b:

$$\begin{aligned} d_z &= d - 2t_f = 18.24 - 2 \times 0.695 = 18.10 \quad (\text{from girder}) \\ w_z &= 14.04 - 2 \times 0.720 = 12.50 \quad (\text{from column}) \end{aligned}$$

$$(d_z + w_z) / 90 = 0.34 < 0.63" \quad \text{OK}$$

Continuity plates

Provide continuity plates per SEAOC 4F2c

Figure D-3. Continued.